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REPORT
on
GEOTECHNICAL INVESTIGATION

PROPOSED COMMUNITY HEALTH CENTRE
103 - 105 REDFERN STREET
REDFERN

Prepared for
ATKINSON CAPITAL INSIGHT

Project 44897
July 2007

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Project 44897
18 July 2007

REPORT ON GEOTECHNICAL INVESTIGATION
PROPOSED COMMUNITY HEALTH CENTRE
103 - 105 REDFERN STREET, REDFERN

1. INTRODUCTION

This report presents the results of a geotechnical investigation undertaken for a proposed community health centre at 103 - 105 Redfern Street, Redfern. The work was carried out at the request of Atkinson Capital Insight.

It is understood that the proposed development includes construction of a three-storey building with a single level of basement carparking to the south of the existing heritage-listed courthouse. The basement floor level will require excavation to depths up to 1.0 m to 2.8 m. The investigation was carried out to provide information on the subsurface conditions for planning of site works including excavation, support and footings.

The field work for the current investigation comprised three boreholes with in-situ testing and sampling of the subsurface strata followed by engineering analysis and reporting. Details of the field work are given in the report together with comments on design and construction practice.

2. BACKGROUND

Rickard Hails Moretti Pty Ltd (RHM) previously carried out an investigation for the heritage listed courthouse building and prepared a "Geotechnical Engineers Report" (dated July 2006). The investigation included one test pit (at the north-western corner of the building) to expose the existing footing and five penetrometer tests to depths ranging from 1.55 m to 2.2 m.

The test pit exposed a 400mm thick brick wall supported on a concrete strip footing 250mm thick and 950 mm wide. The base of the concrete strip footing was founded on red brown clay at a depth of 0.93 m below existing ground surface. The clay at the base of footing was assessed to have a high moisture content. Water seepage was observed in the base of the pit prior to backfilling. Reference should be made to the RHM report for further details on the investigation and results.

3. SITE DESCRIPTION

The site of the proposed new building is located between a heritage-listed, sandstone and brick courthouse to the north and Turner Street to the south. It is a rectangular-shaped area of approximately 820 m^2 with a 27 m southern frontage to Turner Street. Ground surface levels fall to the south-east at approximately 2° to 3° from approximately RL 38.8 to RL 37.9 relative to Australian Height Datum (AHD).

At the time of the investigation, the site of the proposed new building was occupied by a one and two-storey brick building (Old Redfern Police Station).

One to three-storey brick buildings extend up to, or are set back approximately 1 m from the common boundaries on the properties to the east and west of the site.

4. GEOLOGY

Reference to the Sydney 1:100,000 Geological Series Sheet indicates that the site is underlain by Quaternary sediments comprising medium to fine grained marine sands and that the area to the west of the site (i.e. west of Regent Street) is underlain by Ashfield Shale of the Wianamatta Group. The field work identified residual soils overlying shale which is consistent with the weathered Ashfield Shale profile. Ashfield Shale typically weathers to form clays of medium to high plasticity.

5. FIELD WORK METHOD

The field work for the investigation comprised a site inspection by a senior geotechnical engineer, one borehole (BH 1) drilled to a depth of 5.0 m using a Bobcat-mounted drilling rig and two hand augered boreholes (BH 2 and BH 3) drilled to refusal at depths of 0.9 m.

At each of the hand augered borehole locations dynamic cone penetrometer (DCP) tests were carried out to practical refusal at depths of 1.8 m to provide an indication of the soil strength and probe the depth to weathered shale. A standard penetration test (SPT) was carried out within BH 1 at a depth of 1.0 m to sample the soil and assess the in-situ strength of the material. Disturbed samples were also taken at regular depth intervals.

The boreholes were sampled and logged on site by an experienced geologist. Observations of soil moisture conditions were made during auger drilling of the boreholes, which were backfilled immediately on completion of drilling, precluding longer term monitoring of groundwater.

The ground surface levels at each of the bore locations were interpolated from spot heights relative to Australian Height Datum (AHD) shown on the survey plan by Lockley Land Title Solutions Pty Ltd (Job Reference 29651DT, dated 21/7/06).

6. FIELD WORK RESULTS

Details of the conditions encountered are given in the borehole logs in Appendix A, together notes defining classification methods and descriptive terms. The borehole locations are shown on Drawing 1 in Appendix B.

The boreholes penetrated a subsurface profile comprising pavements and filling overlying residual clay then shale. The various strata are summarised below and a geological section (Section A-A') through the site is given on Drawing 2 in Appendix B.

PAVEMENTS - comprising asphaltic concrete or concrete 100 mm to 120mm thick over a 50 mm to 100 mm thick layer sandy gravel filling (roadbase).

FILLING - comprising sandy clay to a depth of 0.3 m in BH 2 and sand to a depth of 0.4 m in BH 3.

RESIDUAL CLAY - typically stiff to very stiff clay to depths of 0.6 m to 0.9 m over very stiff shaly clay to depths of 1.5 m to 1.8 m.

SHALE - encountered at a depth of 1.5 m (RL 36.3) in BH 1. The shale was initially extremely low to very low strength to a depth of 3.2 m (RL 34.6) then low strength shale with medium strength ironstone bands. DCP 2 and DCP 3 encountered refusal on inferred shale at depths of 1.8 m (RL 36.6 and RL 37.2 respectively).

Groundwater was not encountered during auger drilling within the boreholes.

7. GEOTECHNICAL MODEL

The inferred geological model for the site comprises:

- a surficial zone of pavements and filling up to approximately 0.4 m deep.
- stiff to very stiff residual clay to depths of approximately 0.6 m to 0.9 m then very stiff shaly clay to depths of 1.5 m to 1.8 m. The shaly clay extends to approximately 0.6 m

below the southern side of the basement floor level. Experience in the area indicates the clays are of high plasticity and highly reactive with changes in moisture content.

- shale bedrock (Ashfield Shale) underlying the residual clay at depths of 1.5 m to 1.8 m. It is anticipated that the extremely low strength shale may be exposed or just below the north-western section of the basement.
- an intermittent perched groundwater table on the top of the clay and/or rock surface, particularly following periods of extended wet weather.

8. COMMENTS

8.1 Proposed Development

The drawings by Atkinson Capital Insight (Drawing No. DA101 to 107, December 2006) indicate that the proposed development includes construction of a three-storey building with a single level of basement carparking to the south of the existing heritage-listed courthouse. The basement floor level (RL 36.97) will require excavation to depths ranging from approximately 1.0 m to 2.8 m (increasing in depth toward the north-western corner). The basement will be set back approximately 2 m and 1.5 m from the eastern and western boundaries, respectively and will extend up to the southern boundary and the courthouse (to the north). A ramp will enter the carpark from Turner Street.

The outline of the proposed structure is shown on Drawings 1 and 2.

8.2 Excavation Conditions

As shown on Drawing 2, it is anticipated that bulk excavation to depths of 1 m to 2.8 m will involve removal of filling, residual clay and possibly extremely low strength to very low strength shale toward the north-western corner of the basement.

Excavation of soil and extremely low strength shale should be readily achieved using conventional earthmoving equipment. Excavation of very low strength shale or thick ironstone bands within the clay and shale profile may require light to moderate ripping or possibly rock hammering with an impact breaker mounted on a hydraulic excavator for effective excavation. It is anticipated that the proposed excavation will generally result in relatively minor vibrations.

If impact breakers are required then it would be prudent to monitor and limit vibrations on adjacent properties, in particular for the heritage listed courthouse. Generally, a maximum peak particle velocity of 8 mm/sec (any component direction) at foundation level of adjacent structures should be adopted for both structural and human comfort considerations. It is suggested that this should be limited to 5 mm/sec (any component direction) for the courthouse. A vibration monitoring trial may be carried out in order to establish a site specific vibration attenuation relationship and safe operating distances for various earthmoving and excavating equipment.

Groundwater seepage was not encountered within the boreholes for the current investigation, however some seepage may be expected along the top of the clay and rock surface, particularly following periods of extended wet weather. Seepage was observed close to the natural clay surface during the previous investigation by RHM. Such seepage should be readily controlled by perimeter drains to a "sump-and-pump" dewatering system.

8.3 Disposal of Excavated Material

Under the Waste Avoidance and Resource Recovery Act (NSW EPA, 2001) a waste/fill receiving site must be satisfied that materials received meet the environmental criteria for proposed land use. This includes filling and virgin excavated natural materials (VENM), such as may be removed from this site. Accordingly, environmental testing will need to be carried out to classify any spoil which is to be removed from the site.

8.4 Dilapidation Surveys

It is suggested that dilapidation (existing condition) reports should be undertaken on the adjacent buildings. As a minimum this should include the buildings to the west and courthouse to the north of the proposed excavation. The condition survey should be undertaken prior to commencing work on the site to document any existing defects so that any claims for damage due to excavations or other construction related activities can be accurately assessed. The owner should be provided with copies of the relevant reports and asked to confirm that they represent a fair and accurate record of the existing condition of the buildings.

8.5 Excavation Support

The basement will be set back approximately 2 m and 1.5 m from the eastern and western boundaries, respectively, and will extend up to the southern boundary and the courthouse (to the north).

During bulk excavation, temporary batter slopes should be battered at no steeper than 1 Horizontal (H) : 1 Vertical (V) in soils and extremely low to very low strength shale. It is anticipated that temporary batters may be achievable along the eastern boundary whilst some form of shoring will be required along the remaining boundaries. The courthouse footings, understood to comprise concrete strip footings founded on residual soils at a depth of approximately 0.9 m, will need to be appropriately supported by shoring or alternatively underpinning could be adopted.

Shoring may comprise a bored soldier pile wall with shotcrete or timber infill panels. Shoring piles may be used to carry vertical structural loads and should be socketed below the bulk excavation to at least 0.3 m into shale of at least extremely low top very low strength.

Underpinning, if adopted for support of the Courthouse, could be carried out using "hit and miss" panel construction techniques. Generally, every fourth panel would be initially excavated and constructed after which the panels are progressively in-filled to form continuous underpinning panels below the entire wall. Panel widths would generally need to be limited to about 1 m to 2 m (depending on the type and condition of the existing footings). Underpinning panels may comprise mortared brickwork but mass concrete is usually easier to place. It should be noted that partial underpinning of the courthouse will probably result in future differential settlement and shrink/well movements between the underpinned edge of the Courthouse (which may be founded on shale in some sections) and the remaining section of the building which is supported on reactive clays at a higher level. Further investigation of the existing footings along the southern side of the Courthouse should be carried out to determine the most appropriate underpinning procedure and provide further assessment on the potential differential movements that may occur.

Retaining walls and/or underpinning panel sections may be designed on the basis of an average unit weight of 20 kN/m^3 for soil and a triangular earth pressure distribution based on an active earth pressure coefficient (Ka) of 0.3 where some wall movement is acceptable. An at-rest earth pressure coefficient (Ko) of 0.5 should be used where wall movement is to be minimised, such as close to existing structures or where the wall is propped or braced.

An allowable lateral restraint equal to 200 kPa may be adopted within shale of at least extremely low to very low strength which is at least 0.3 m below the adjacent bulk excavation level.

Drainage of the ground behind impermeable walls should be provided otherwise the wall should be designed for full hydrostatic pressures. Drainage could comprise 150 mm wide strip drains pinned to the face at 2 m centres behind shotcrete in-fill panels. The base of the strip drains should extend out from the shoring wall to allow any seepage to flow into a perimeter toe drain which is connected to a sump dewatering system.

All surcharge loads should be allowed for in the retaining wall design including building footings, traffic and construction related activities.

8.6 Foundations

It is anticipated that extremely low to very low strength shale will be exposed within the deeper parts of the excavation toward the north-western corner of the basement. It is suggested that all structural loads should be uniformly supported on shale of extremely low to very low strength, for which conventional pad and strip footings should generally be appropriate. Where only minor excavation is carried out it may be necessary to use deepened pad footings. Alternatively, bored piers may be required if the depth to shale is greater than about 1 m to 1.5 m.

Footings founded on shale of at least extremely low strength may be designed on this basis of an allowable end bearing pressure of 700 kPa and an allowable shaft adhesion value of 70 kPa.

Alternatively, bored piers may be used to found on the low strength shale which was encountered at a depth of 3.2 m in BH 1. Bored piers founded on shale of at least low strength or better may be designed on this basis of an allowable end bearing pressure of 1500 kPa and an allowable shaft adhesion value of 150 kPa.

Footings designed using the above parameters could expect settlements of up to 1% of the footing width.

Differential settlements between the new building and existing Courthouse are inevitable. Hence, the design should allow for detailing of finishes to avoid future trip hazards and minor cracking.

All footings should be inspected by a geotechnical engineer to confirm that founding material is suitable for the design bearing pressures and shaft adhesion values assumed in the design.

DOUGLAS PARTNERS PTY LTD

Reviewed by

Scott Easton
Associate

Grahame Wilson
Principal

APPENDIX A

Notes Relating to this Report
Results of Field Work

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NOTES RELATING TO THIS REPORT

Introduction

These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the Discussion and Comments section. Not all, of course, are necessarily relevant to all reports.

Geotechnical reports are based on information gained from limited subsurface test boring and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigations Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00 mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Undrained Shear Strength kPa
Very soft	less than 12
Soft	12—25
Firm	25—50
Stiff	50—100
Very stiff	100—200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT "N" Value (blows/300 mm)	CPT Cone Value (q_c — MPa)
Very loose	less than 5	less than 2
Loose	5—10	2—5
Medium dense	10—30	5—15
Dense	30—50	15—25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing with a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling are given in the report.

Drilling Methods.

The following is a brief summary of drilling methods currently adopted by the Company and some comments on their use and application.

Test Pits — these are excavated with a backhoe or a tracked excavator, allowing close examination of the in-situ soils if it is safe to descent into the pit. The depth of penetration is limited to about 3 m for a backhoe and up to 6 m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) — the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling — the hole is advanced by pushing a 100 mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers — the hole is advanced using 90—115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling — the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling — similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. from SPT).

Continuous Core Drilling — a continuous core sample is obtained using a diamond-tipped core barrel, usually 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" — Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of say 4, 6 and 7

  as 4, 6, 7
  N = 13

- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm

  as 15, 30/40 mm.

The results of the tests can be related empirically to the engineering properties of the soil.

Occasionally, the test method is used to obtain samples in 50 mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch cone — abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australian Standard 1289, Test 6.4.1.

In the tests, a 35 mm diameter rod with a cone-tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20 mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: —

- Cone resistance — the actual end bearing force divided by the cross sectional area of the cone — expressed in MPa.
- Sleeve friction — the frictional force on the sleeve divided by the surface area — expressed in kPa.
- Friction ratio — the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0—5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0—50 MPa) is less sensitive and is shown as a full line.

The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1%—2% are commonly encountered in sands and very soft clays rising to 4%—10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range:—

$$q_c \text{ (MPa)} = (0.4 \text{ to } 0.6) \text{ N (blows per 300 mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:—

$$q_c = (12 \text{ to } 18)\ c_u$$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculation of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150 mm increments of penetration. Normally, there is a depth limitation of 1.2 m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer — a 16 mm diameter flat-ended rod is driven with a 9 kg hammer, dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as the Scala Penetrometer) — a 16 mm rod with a 20 mm diameter cone end is driven with a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). The test was developed initially for pavement subgrade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is carried out in accordance with Australian Standard 1289 "Methods of Testing Soil for Engineering Purposes". Details of the test procedure used are given on the individual report forms.

Bore Logs

The bore logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable, or possible to justify on economic grounds. In any case, the boreholes represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than 'straight line' variations between the boreholes.

Ground Water

Where ground water levels are measured in boreholes, there are several potential problems;

- In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building), the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface condition, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- unexpected variations in ground conditions — the potential for this will depend partly on bore spacing and sampling frequency
- changes in policy or interpretation of policy by statutory authorities
- the actions of contractors responding to commercial pressures.

If these occur, the Company will be pleased to assist with investigation or advice to resolve the matter.

Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some later stage, well after the event.

Reproduction of Information for Contractual Purposes

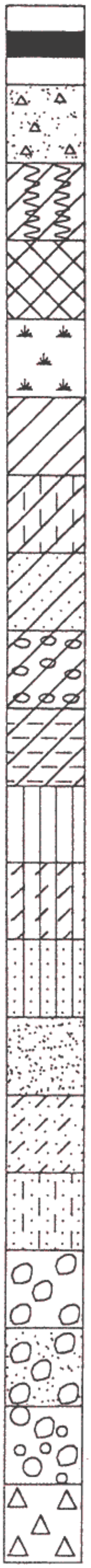
Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

GRAPHIC SYMBOLS FOR SOIL & ROCK

SOIL

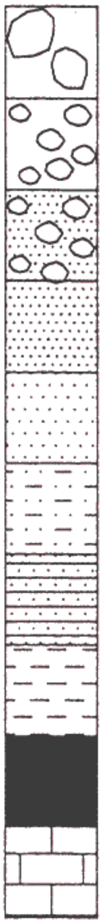


- BITUMINOUS CONCRETE
- CONCRETE
- TOPSOIL
- FILLING
- PEAT
- CLAY
- SILTY CLAY
- SANDY CLAY
- GRAVELLY CLAY
- SHALY CLAY
- SILT
- CLAYEY SILT
- SANDY SILT
- SAND
- CLAYEY SAND
- SILTY SAND
- GRAVEL
- SANDY GRAVEL
- COBBLES/BOULDERS
- TALUS

SEAMS

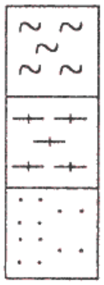
- SEAM >10mm
- SEAM <10mm

SEDIMENTARY ROCK



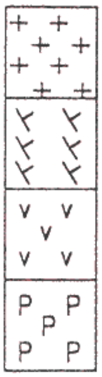
- BOULDER CONGLOMERATE
- CONGLOMERATE
- CONGLOMERATIC SANDSTONE
- SANDSTONE FINE GRAINED
- SANDSTONE COARSE GRAINED
- SILTSTONE
- LAMINITE
- MUDSTONE, CLAYSTONE, SHALE
- COAL
- LIMESTONE

METAMORPHIC ROCK



- SLATE, PHYLLITE, SCHIST
- GNEISS
- QUARTZITE

IGNEOUS ROCK



- GRANITE
- DOLERITE, BASALT
- TUFF
- PORPHYRY

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Geotechnics, Environment, Groundwater

BOREHOLE LOG

CLIENT: Atkinson Capital Insight
PROJECT: Proposed Community Health Centre
LOCATION: 103-105 Redfern Street, Redfern

SURFACE LEVEL: 37.8 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 44897
DATE: 26 May 07
SHEET 1 **OF** 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Well Construction Details
37.8		ASPHALTIC CONCRETE							
	0.1	FILLING - grey sandy gravel (roadbase)							
	0.15	CLAY - stiff to very stiff, mottled orange brown clay, damp		A	0.3 – 0.5				
	0.6	SHALY CLAY - very stiff, mottled red/grey shaly clay with a trace of ironstone		A					
36.8	1			S	1.0 – 1.45		3,10,15 N = 25		1
	1.5	SHALE - extremely low to very low strength, light grey shale with ironstone bands		A	1.5 – 2.0				
35.8	2								2
34.8	3								3
	3.2	SHALE - low strength, grey to grey brown shale, with medium strength ironstone bands		A	3.2 – 3.5				
33.8	4								4
				A	4.5 – 5.0				
32.8	5.0	Bore discontinued at 5.0m							5

RIG: Bobcat **DRILLER:** Trippet **LOGGED:** SI **CASING:** Uncased
TYPE OF BORING: Diatube to 0.1m; Hand auger to 0.5m; Solid flight auger to 5.0m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: [signature]
Date: 7/07

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BOREHOLE LOG

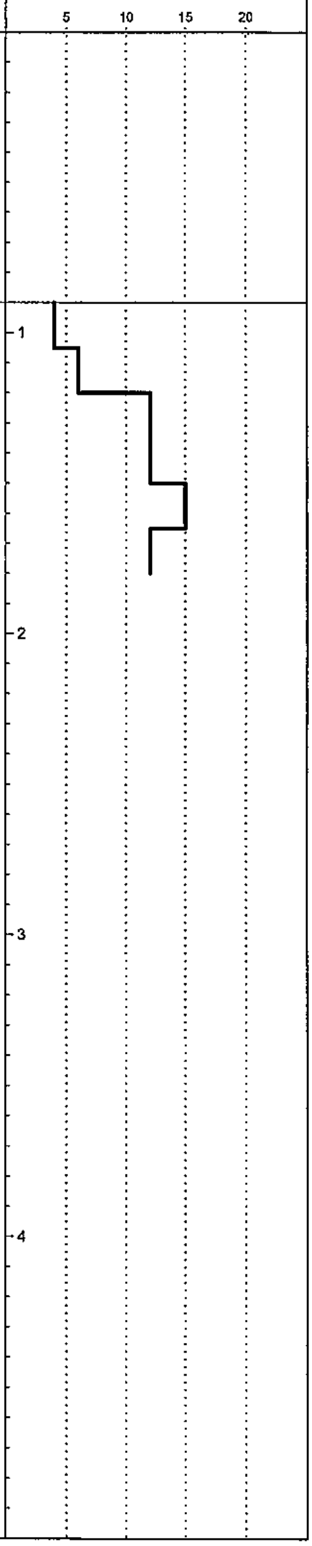
CLIENT: Atkinson Capital Insight
PROJECT: Proposed Community Health Centre
LOCATION: 103-105 Redfern Street, Redfern

SURFACE LEVEL: 38.4 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 44897
DATE: 26 May 07
SHEET 1 **OF** 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
38.4		FILLING - grey to dark grey, fine to medium grained, sand filling with wood chips						
	0.15	FILLING - grey brown sandy clay filling						
	0.3	CLAY - stiff, mottled orange brown clay, damp		A	0.3–0.5			
	0.7	SHALY CLAY - very stiff, mottled grey and red brown shaly clay with ironstone bands		A	0.7–0.9			
	0.9	Bore discontinued at 0.9m - refusal on ironstone band						
37.4	1							
36.4	2							
35.4	3							
34.4	4							

Dynamic Penetrometer Test (blows per 150mm)



RIG: Hand tools **DRILLER:** SI **LOGGED:** SI **CASING:** Uncased

TYPE OF BORING: Hand auger to 0.9m; DCP to 1.8m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS:

☐ Sand Penetrometer AS1289.6.3.3
☐ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ⊻ Water level

CHECKED
Initials: [signature]
Date: 7/07

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BOREHOLE LOG

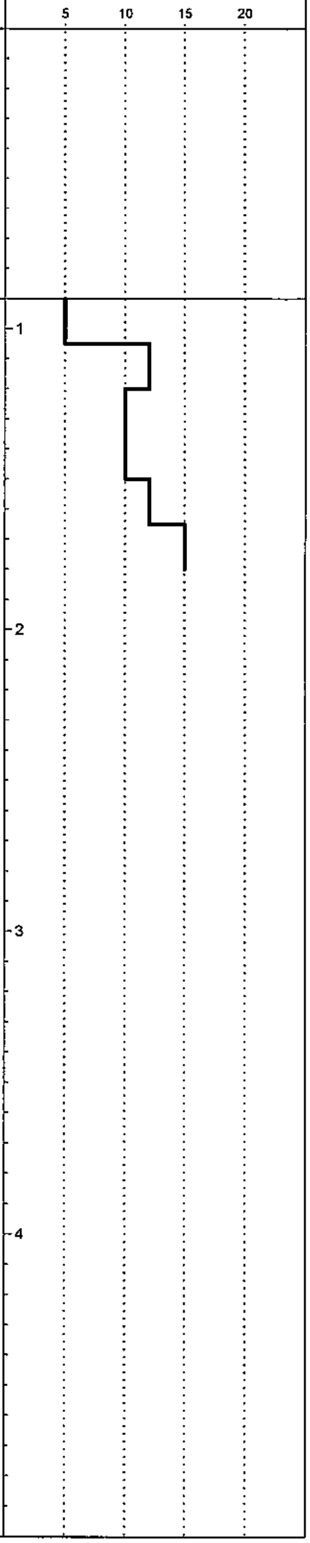
CLIENT: Atkinson Capital Insight
PROJECT: Proposed Community Health Centre
LOCATION: 103-105 Redfern Street, Redfern

SURFACE LEVEL: 39.0 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 44897
DATE: 26 May 07
SHEET 1 **OF** 1

RL	Depth (m)	Description of Strata	Graphic Log	Type	Depth	Sample	Results & Comments	Water
39.0		CONCRETE - 120mm thick						
	0.12	FILING - grey sandy gravel (roadbase)						
	0.2	FILLING - brown medium grained, sand filling						
	0.4	CLAY - stiff, mottled orange brown clay with ironstone gravel		A	0.5			
				A	0.9			
38.0	0.9	Bore discontinued at 0.9m - refusal on ironstone band						
37.0	2							
36.0	3							
35.0	4							

Dynamic Penetrometer Test (blows per 150mm)



RIG: Hand tools **DRILLER:** SI **LOGGED:** SI **CASING:** Uncased
TYPE OF BORING: Hand auger to 0.9m; DCP to 1.8m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

☐ Sand Penetrometer AS1289.6.3.3
☐ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ⊻ Water level

CHECKED
Initials: [signature]
Date: 7/07

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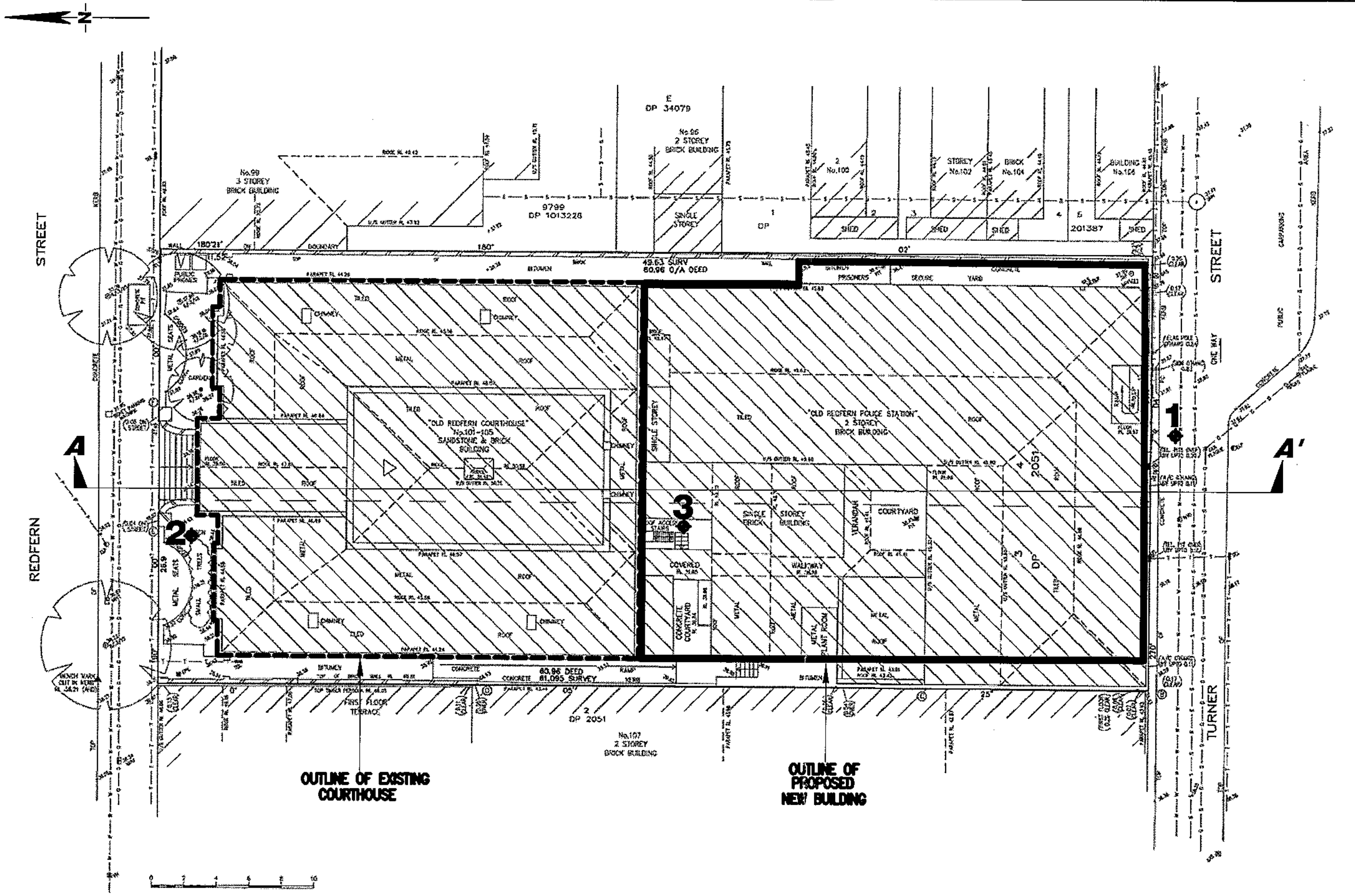
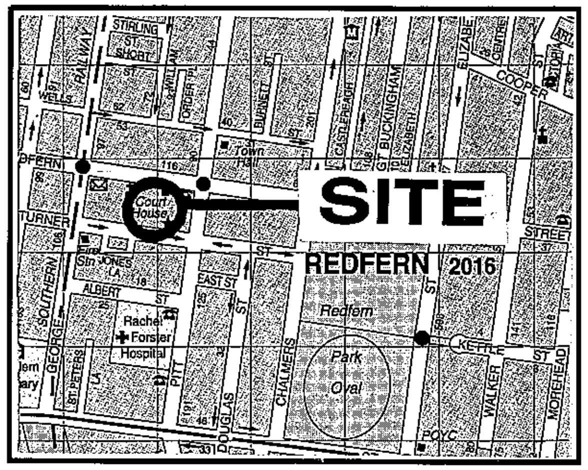
APPENDIX B

Drawing 1 - Location of Tests
Drawings 2 - Geological Cross Section A-A'





LOCALITY PLAN

LEGEND

◆ TEST BORE LOCATION

REFER TO DRAWING 2
FOR GEOLOGICAL SECTION A–A'

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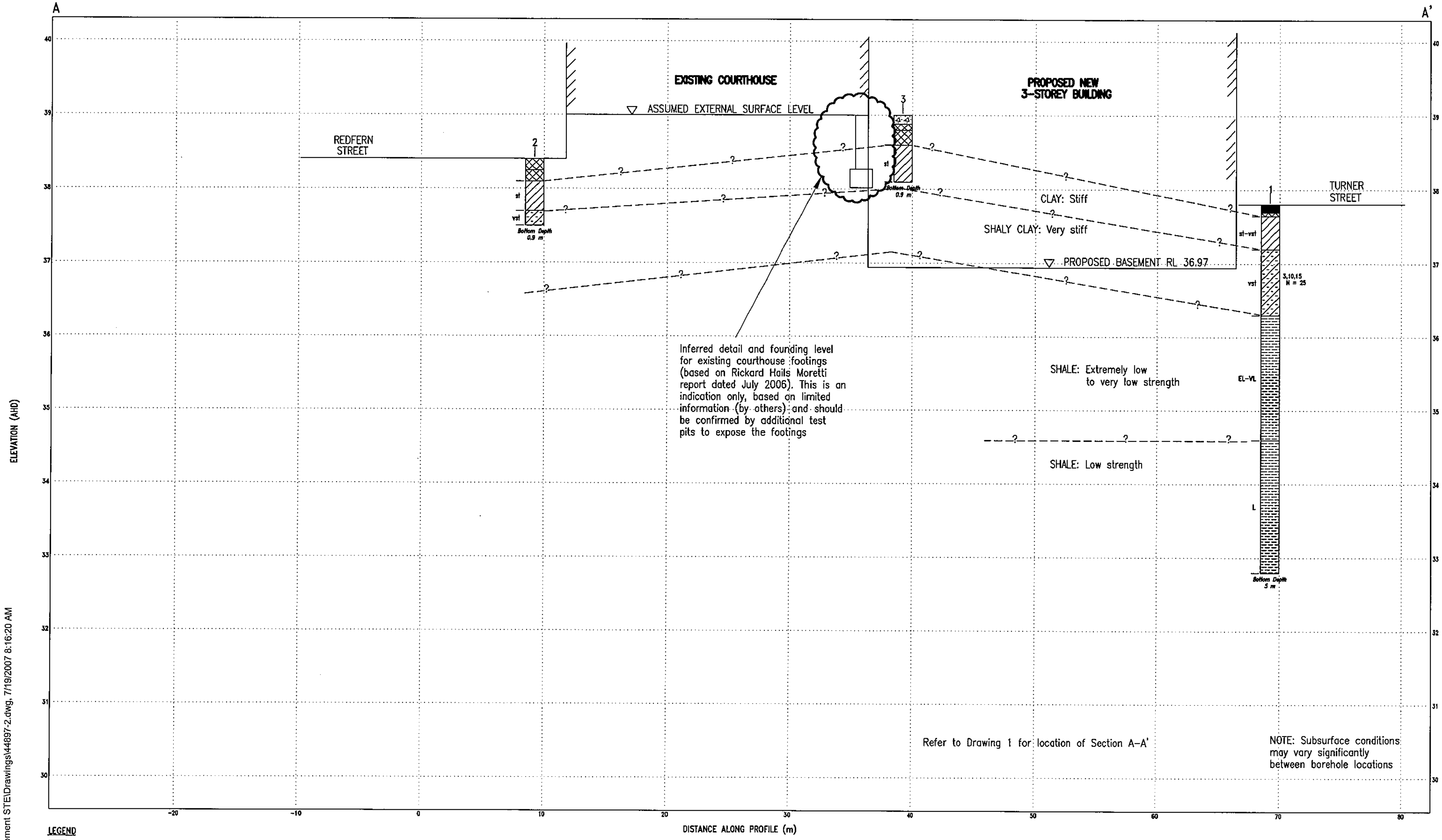
Sydney, Newcastle, Brisbane, Melbourne, Wyong, Canberra, Campbelltown, Townsville, Perth, Cairns, Wollongong, Darwin, Gold Coast, Sunshine Coast

TITLE:
Location of Tests
Proposed Community Health Centre
103-105 Redfern Street
REDFERN

CLIENT: Atkinson Capital Insight			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 44897	OFFICE: SYDNEY
APPROVED BY:		DATE: 12.7.2007	DRAWING No: 1

P:\44897 REDFERN, proposed development STE\Drawings\44897-1.dwg, 7/12/2007 8:05:54 AM



LEGEND

- Bituminous Concrete
- Filling
- Clay
- Shaly Clay
- Shale
- Concrete

ROCK STRENGTH
EL – Extremely Low
VL – Very Low
L – Low

SOIL CONSISTENCY
st – stiff
vst – very stiff

TESTS / OTHER
N – Standard penetration test value

Vertical Scale (metres) 0–1
Horizontal Scale (metres) 0–6
NOTE: Exaggerated vertical scale & horizontal scale

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CLIENT: Atkinson Capital Insight			TITLE: Section A-A' 103-105 Redfern Street, Redfern
PROJECT: Proposed Community Health Centre			
APPROVED BY: STE	DATE: 12 July 07	REVISION:	SCALE: 1: 300 (H)
PROJECT No: 44897	DRAWING No: 2		

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